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A Design of an Armory

Architectural Engineering

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A DESIGN OF AN ARMORY

BY

JESSE RAYMOND WHEATON

THESIS

FOR THE

DEGREE OF BACHELOR OF SCIENCE

IN

ARCHITECTURAL ENGINEERING

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

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May 29 1912

THIS IS TO CERTIFY THAT THE THESIS PREPARED UNDER MY SUPERVISION BY

Jesse Raymond Wheaton

ENTITLED A DESIGN OF AN ARMORY

IS APPROVED BY ME AS FULFILLING THIS PART OF THE REQUIREMENTS FOR THE

DEGREE OF Bachelor of Science in

Architectural Engineering

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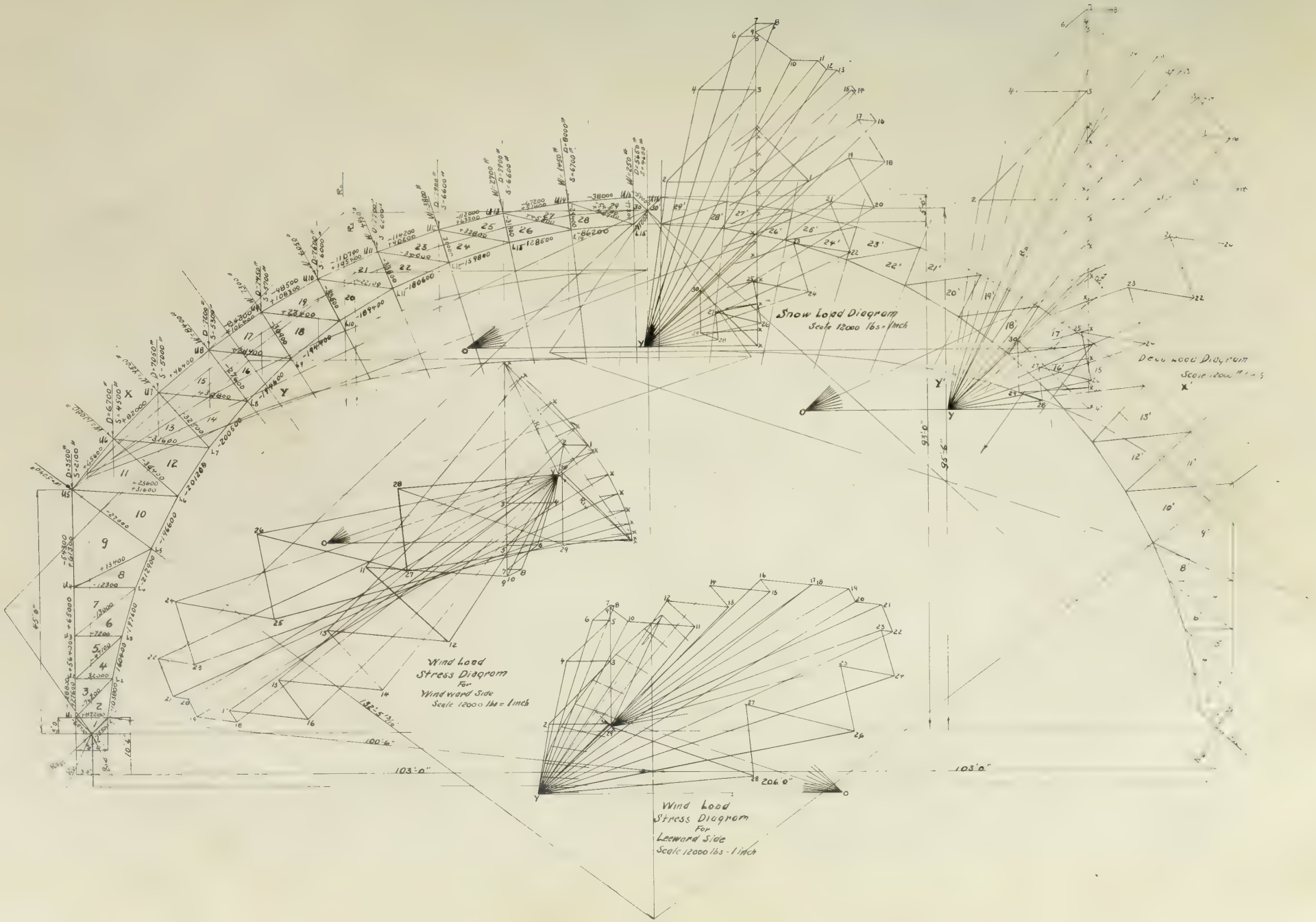
Snow Load Diagram
Scale 12000 lbs = 1 inch
Wind Load
Stress Diagram
For
Windward Side
Scale 12000 lbs = 1 inch
Wind Load
Stress Diagram
For
Leeward Side
Scale 12000 lbs = 1 inch
103'-0"
103'-0"
206'-0"

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A DESIGN OF AN ARMORY

The object of this thesis is to design a practical and economical armory to be erected at the University of Illinois. The main structural framing of this building consists of fourteen (14) steel arch trusses of the 3-hinged-type. The general design and some details are shown on accompanying cuts. The space of the arches center to center of piers is 206 feet with a rise from the finished floor line to the upper pin of 94 feet, the finished floor being 1 foot 6 inches above the center line of the end pins. The upper chord panel points are on the circumference of a circle whose radius is 132 feet $5\frac{13}{16}$ inches, with the center located 36 feet 3 inches below the floor line. For economy in shop work, this chord member is divided into ten (10) equal panels each 12 feet in length. The lower chord panel points are on the circumference of a circle whose radius is 100 feet 6 inches the center being 9 feet below the

floor line. The chord members consist of a box section composed of two (2) channels laced, with the webs horizontal for ease in bending at the panel points. The web members consist of either two (2) or four (4) angles laced. The arches are placed 28 feet apart, and are connected transversly by eleven (11) lines of lattice girders. These arches have 5½ inch shoe pins and a 4 inch crown pin. The shoe pins of each truss are connected by a tie consisting of a 6 by ¾ inch eye bar. Particular care has been taken in this design to make the $\frac{L}{R}$ in both directions nearly the same.

Wind Loads

The roof is designed for a wind load of 30 pounds per square foot on a vertical surface. There are many formulas that may be used, but the straight line formula is preferred by many on account of its simplicity, and gives results which agree closely with experiments. It will therefore be adopted in connection with this design: formula, $P_n = \frac{P}{45} A$. Where P_n is the normal component of the wind pressure, P is the pressure per square foot on a vertical surface; A is the angle which the roof surface makes with the horizontal in degrees, except where the angle is more than 45 degrees. In that case,

$P_n = P$

A pressure P per square foot on a vertical surface is usually taken at about 30 pounds per square foot, which is equivalent to a wind velocity of 87 miles per hour. Professor Malcolm says that this assumed pressure is sufficient for all except the most exposed positions. Consequently a pressure P of 30 pounds per square foot will

be used in this design. The wind pressure on the sides of the building is not figured in with the wind pressure on the roof as this is taken care of by a wall and by inside abutments which do not come in contact with the steel arches in any way.

The following table gives the calculations as figured from the preceding formula for the wind loads at the different panel points.

Calculations

Table Of Wind Loads

Panel Number	$\frac{Pn}{45}$	Angle Degrees	Width In feet	Pounds per □'	Pounds per 28' Span
1	.30	50	6	180	5040
2	$\frac{30}{45}$ = .66	48	12	360	10080
3	.66	42.5	12	340	9500
4	.66	37.5	12	300	8400
5	.66	32.5	12	260	7300
6	.66	27	12	216	6050
7	.66	22	12	176	4950
8	.66	17	12	136	3800
9	.66	12	12	96	2700
10	.66	6.5	12	52	1450
11	.66	1.5	8.5	8.5	250

Snow Loads

The amount of the snow load to be considered varies greatly for different localities. The weight of the snow, which must be taken into account, is a function of both the latitude and the humidity, and these factors should be carefully considered in determining the weight to be used for any particular locality. The maximum wind load and the maximum snow load will probably never occur at the same time, and if the maximum wind is taken, then a smaller snow load should be used.

For a latitude of about 35 to 40 degrees, Professor Malcolm recommends a minimum snow load of 10 pounds and a maximum snow load of 20 pounds per square foot of the horizontal projection of the roof. Hence, a maximum snow load of 20 pounds per square foot will be adopted. To obtain the snow loads at the respective panel points, multiply the horizontal projection of the roof surface carried at each panel points by the snow load per square foot and by the spacing of trusses.

TABLE OF SNOW LOADS

Panel No	Hor proj roof Surface feet	Snow Load per sq ft lbs	Load per Sq. ft. lbs	Load per 28 ft Span lbs
1st "	3.75	20	75	2100
2nd "	8	20	165	4500
3rd "	9	20	180	5000
4th "	9.5	20	190	5300
5th "	10.25	20	205	5700
6th "	10.75	20	215	6000
7th	11	20	220	6200
8th	11.75	20	235	6600
9th "	11.75	20	235	6600
10th "	12	20	240	6700
11th "	8.5	20	165	4600

Dead Loads

In connection the dead loads for the arches, instead of assuming 20 or 30 pounds per square foot of horizontal projection as is sometimes done, the loads will be deduced from the results of the investigations by Doctor N. Clifford Ricker of the University of Illinois, who is one of the best authorities on the design of roofs in this country. He worked out the following formula which was found to agree more closely with the computed weights of the trusses examined than formulas previously used by the majority of designers: $W = \frac{S}{25} + \frac{S}{12\,600}$

S = the span in feet

W = the weight of the truss in pounds per square foot of horizontal projection of the roof. In the case at hand, the span, S = 206'-0".

Therefore $W = \frac{206}{25} + \frac{206}{12\,600} = 8.25 + \frac{42\,436}{12\,600} = 8.25 + 3.37$

= 11.62 pounds per square foot of horizontal projection of roof surface.

The calculations below show the weight of truss per square foot of horizontal projection of the roof at each panel point and also copper roofing, sheathing, purlins, wind bracings and rafters.

TABLE OF DEAD LOADS

Panel No.	Weight truss per □ ft lbs	PANEL Width ft.	PANEL length ft.	Load at panel pt. due to Wt. of truss. lbs.	Wt rafters purlin & bracing lbs	Total Load lbs
1	11.62	3.75	28	1220	2490	3800.
2	11.62	8	28	2600	4380	7000
3	11.62	9	28	2950	4380	7350
4	11.62	9.5	28	3150	4380	7500
5	11.62	10.25	28	3350	4380	7650
6	11.62	10.25	28	3500	4380	7900
7	11.62	11	28	3600	4380	8000
8	11.62	11.75	28	3800	4380	8200
9	11.62	11.75	28	3800	4380	8200
10	11.62	12	28	3900	4380	8300
11	11.62	8.25	28	2700	3450	6150

TABLE No 4

3 HINGED ARCH STRESSES

Member	Dead Load	Snow Load	Wind Load	D+S Loads	D+W Loads	Maximum
X-1	27700-	20200-	32000-	47900-	59700-	59700-
X'-1"	"	"	1000+	"	26700-	"
X-3	9700+	10400+	38300-	20100+	28800-	28800-
X'-3'	"	"	17900+	"	27600+	27600+
X-5	27000+	24600+	50400-	51600+	23400-	
X'-5'	"	"	29400+	"	56400+	56400+
X-7	31600+	28400+	56800-	60000+	25200-	
X'-7'	"	"	33400+	"	65000+	65000+
X-9	28500+	25800+	58800-	54300-	30300-	54300-
X'-9'	"	"	32800+	"	61300+	61300+
X-11	29400+	26000+	68000-	55400+	38600-	
X'-11'	"	"	36000+	"	65600+	65600+
X-13	35800+	31200+	88000-	67000+	52200-	
X'-13	"	"	46200+	"	82000+	82000+
X-15	39200+	33800+	107300-	73000+	68100-	
X'-15'	"	"	57200+	"	96400+	96400+
X-17	38900+	33600+	123200-	72500+	84300-	84300-
X'-17'	"	"	67400+	"	106400+	106400+
X-19	32900+	39000+	131400-	71900+	98500-	98500-
X'-19'	"	"	75400+	"	108300+	108300+
X-21	24000+	21600+	134700-	45600+	110700-	110700-
X'-21'	"	"	81400+	"	105400+	105400+
X-23	11600+	10700+	125800-	22300+	114200-	114200-
X'-23'	"	"	78600+	"	90200+	90200+
X-25	2000-	1200-	110000-	3200-	112000-	112000-
X'-25'	"	"	65200+	"	63200+	63200+
X-27	13800-	11600-	62400-	25400-	76200-	76200-
X'-27'	"	"	37600+	"	51400+	51400+
X-29	19000-	15500-	19000-	34500-	38000-	38000-
X'-29'	"	"	400-	"	19400-	"
X-30	27000-	22000-	27000-	49000-	54000-	54000-
X'-30'	"	"	700-	"	27700-	"
Y-1	91000-	64000-	11400-	155000-	102400-	155000-
Y'-1'	"	"	27650-	"	118650-	"
Y-2	58000-	45800-	8000-	103800-	66000-	103800-
Y'-2'	"	"	19600-	"	77600-	"
Y-4	88700-	71700-	8000+	160400-	80700-	160400-
Y'-4'	"	"	37500-	"	126200-	"
Y-6	10900-	88600-	20400+	197600-	88600-	197600-
Y'-6'	"	"	50400-	"	159400-	"
Y-8	117700-	95200-	28200+	212900-	89500-	212900-
Y'-8'	"	"	56200-	"	173900-	"

TABLE NO 4

Member	Dead Load	Snow Load	Wind Load	D+S Loads	D+W Loads	Maximum
Y-10	108600-	88000-	31500+	196600-	77100-	196600-
Y'-10'	"	"	53800-	"	162400-	"
Y-12	110600-	90600-	55000+	201200-	55600-	201200-
Y'-12'	"	"	64200-	"	174800-	"
Y-14	109900-	90600-	76800+	200500-	33100-	200500-
Y'-14'	"	"	74700-	"	184600-	"
Y-16	106400-	88200-	96600+	194600-	9800-	194600-
Y'-16'	"	"	85400-	"	191800-	"
Y-18	99200-	82600-	112000+	181800-	12800+	
Y'-18'	"	"	95200-	"	194400-	194400-
Y-20	87400-	73400-	119600+	160800-	32200+	
Y'-20'	"	"	102000-	"	189400-	189400-
Y-22	73600-	62000-	121600+	135600-	48000+	
Y'-22'	"	"	107000-	"	180600-	180600-
Y-24	56800-	47300-	111300+	104100-	54500+	
Y'-24'	"	"	103000-	"	159800-	159800-
Y-26	39900-	32800-	84600+	72700-	44700+	
Y'-26'	"	"	88600-	"	128500-	128500-
Y-28	26600-	20500-	44400+	47100-	17800+	
Y'-28'	"	"	59600-	"	86200-	86200-
Y-30	27000-	22000-	1000-	49000-	28000-	
Y'-30'	"	44000-	27600-	71000-	54600-	71000-
1-2	48600+	38600+	6800+	87200+	55400+	87200+
1'-2'	"	"	16600+	"	65200+	"
2-3	41400-	34800-	22500+	76200-	18900-	76200-
2'-3'	"	"	24400-	"	65800-	"
3-4	19200-	15500+	12800-	3700-	32000-	32000-
3'-4'	"	"	12400-	"	31600-	"
4-5	25900-	21200-	17400-	47100-	43300-	47100-
4'-5'	"	"	16800-	"	42700-	"
5-6	5800-	4400+	8400-	1400-	7200-	7200-
5'-6'	"	"	5000+	"	800-	"
6-7	6500-	5800-	10800+	12300-	4300+	
6'-7'	"	"	6500-	"	13000-	13000-
7-8	7000-	5300-	3800-	12300-	10800-	12300-
7'-8'	"	"	1200-	"	8200-	"
8-9	7600+	5800+	4200+	13400+	11800+	13400+
8'-9'	"	"	1300+	"	8900+	"
9-10	15000-	12000-	0	27000-	15000-	27000-
9'-10'	"	"	5900-	"	20900-	"
10-11	7200-	7100-	38800+	14300-	31600-	31600+
10'-11'	"	"	18400-	"	25600-	25600-
11-12	5600-	3200-	30800-	8800-	36400-	36400-
11'-12'	"	"	10400+	"	4800+	"
12-13	2200-	3000-	33800+	5200-	31600-	31600-
12'-13'	"	"	17200-	"	19400-	"

TABLE NO 4

Member	Dead Load	Snow Load	Wind Load	D+S Loads	D+W Loads	Maximum
13-14	10200-	7600-	22300-	17800-	32500-	32500-
13'-14'	"	"	7800+	"	2400-	"
14-15	2000+	1000+	28800+	3000+	30800+	30800+
14'-15'	"	"	16400-	"	14400-	14400-
15-16	14000-	10800-	13600-	24800-	27600-	27600-
15'-16'	"	"	4200+	"	9800-	"
16-17	6900+	4800+	22000+	11700+	28900+	28900+
16'-17'	"	"	14000-	"	7100+	"
17-18	17200-	13700-	3800-	30900-	21000-	30900-
17'-18'	"	"	0	"	17200-	"
18-19	12400+	9800+	11000+	22200+	23400+	23400+
18'-19'	"	"	10200-	"	2200+	"
19-20	18400-	15100-	5600+	33500-	12800-	33500-
19'-20'	"	"	4000-	"	22400-	"
20-21	14900-	12400+	4200+	2500-	10700-	
20'-21'	"	"	7200-	"	22100-	22100-
21-22	18600-	15200-	11100+	33800-	7500-	33800-
21'-22'	"	"	8000-	"	26600-	"
22-23	18200+	15800+	10000-	34000+	8200+	34000-
22'-23'	"	"	3000+	"	21200+	"
23-24	16200-	13800-	17800+	30000-	1600+	30000-
23'-24'	"	"	13000-	"	29200-	"
24-25	17800+	15400-	27700-	2400+	9900-	
24'-25'	"	"	15000+	"	32800+	32800+
25-26	13600-	11600-	24000+	25200-	10400+	
25'-26'	"	"	18300-	"	31900-	31900-
26-27	14900+	13900+	43400-	28800+	28500+	
26'-27'	"	"	30600+	"	45500+	45500+
27-28	9400-	7800-	22800+	17200-	13400+	
27'-28'	"	"	20000-	"	29400-	29400-
28-29	6900+	5000+	47800+	11900+	54700-	54700-
28'-29'	"	"	41800+	"	48700+	48700+
29-30	14600-	12000+	20000+	2600-	17200-	17200-
29'-30'	"	"	"	"	"	"

Unit Stresses

The unit stresses here adopted are those mostly used in general practice for the design of steel structures.

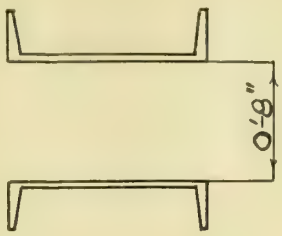
Unit stress tension = 16000 pounds per square inch.
Unit stress compression = $16000 - 70\frac{l}{r}$
Sign of compression = minus (−)
Sign of tension = plus (+)

In chord members having a reversal of stress, add 50 percent of the lesser stress to each stress and design for the stress giving the maximum section. For web members, use $\frac{3}{4}$ of the unit stress per square inch for angles connected by one leg to take care of stress due to eccentricity.

Allowable unit stress in lacing bars = 6000 #/□"
Allowable unit stress in bearing in pins = 15000 #/□"
Allowable unit stress bending = 20000 #/□"
Allowable unit stress for cast iron shoe = 400 #/□"

Design Of Lower Chord Members

For ease in erection, splice the lower chord members at L 6, L 9, L 12, and L 15.



Section L 12 to L 15.

Maximum stress = 159800 pounds compression
Try 2 - 10" ⊏s laced 8" back to back of ⊏s.
Average radius of gyration (r) = 3.7
Length of member (l) = 120"
$\frac{l}{r}$ = 120 ÷ 3.7 = 33.
Allowable unit stress = $P = 16000 - 70\frac{l}{r}$ = 16000 − 70 × 33 = 13700.
Use this for all lower chord members.
Required area = 159800 ÷ 13700 = 11.7 □"
Actual area of 2 - 10" ⊏s 20# = 11.76 □"
Efficiency = 11.76 ÷ 11.7 = 1.01
Use 2 - 10" ⊏s - 20#. Use 2 $\frac{1}{4}$" × $\frac{5}{16}$" lacing bars.

Section L 9 to L 12.

Maximum stress = 194400 pounds compression.
Required area = 194400 ÷ 13700 = 14.2 □"
Try 2 - 10" ⊏s 25#. Actual area = 14.7 □" Use.
Use lacing 2 $\frac{1}{4}$" × $\frac{5}{16}$".

Section L6 to L9.

Maximum stress = 201200 pounds compression.
Required area = 201200 ÷ 13700 = 14.7 □"
Area 2 - 10" [s 25# = 14.7 □" Use
Use 2 1/4" x 5/16" lacing bars

Section L3 to L6.

Maximum stress = 212900 pounds compression.
l = 216" r = 3.7 $\frac{l}{r}$ = 58.
$P = 16000 - 70\frac{l}{r} = 16000 - 58 \times 70 = 12000$# per square inch.
Required area = 212900 ÷ 12000 = 17.6 □"
Try 2 - 10" [s 30#. Area = 17.64 □" Use.
Use 2 1/4" x 5/16" lacing bars.

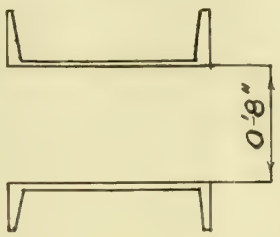
Section L3 to L1.

Maximum stress = 197600 pounds compression.
l = 288" r = 3.7 $\frac{l}{r}$ = 78.
$P = 16500 - 70\frac{l}{r} = 16000 - 70 \times 78 = 10500$# per square inch.
Required area = 197600 ÷ 10500 = 18.9 □"
Try 2 - 10" [s 35#. Area = 20.58 □" Use
Use 2 1/4" x 5/16" lacing bars.

Design Of Upper Chord Members

In chords having a reversal of stress, we will add 50 Percent of the lesser stress to each stress and design for the stress giving the maximum section.

Section U13 to U15.



Maximum stresses compression $-67200^{\#}$ tension $+51460^{\#}$
compression $-25700^{\#}$ tension $+25700^{\#}$
compression $-92900^{\#}$ tension $+77100^{\#}$

For compression, $r = 3.7$ $l = 120$ $\frac{l}{r} = 33$
$P = 13700$ compression. $P = 16000$ tension.
Required area $= -92900 \div 13700 = 6.8\ \square''$
Try 2 - 10" [s $15^{\#}$ Area $= 8.92\ \square''$
Tension $77100 \div 16000 = 4.8\ \square''$ area.
Try 2 - 10" [s $15^{\#}$ Area $= 8.92 - 1.53 = 7.39\ \square''$
Use 2 - 10" [s $15^{\#}$ and $2\frac{1}{4}'' \times \frac{5}{16}''$ lacing bars.

Section U8 to U11.

Maximum stresses compression $-110700^{\#}$ tension $+105400^{\#}$
compression $-52700^{\#}$ tension $+52700^{\#}$
compression $-163400^{\#}$ tension $+158100^{\#}$

Required area $= 163400 \div 13700 = 11.9\ \square''$
Try 2 - 10" [s $20^{\#}$ Area $= 11.76\ \square''$, or $158100 \div 16000 = 9.9$
2 - 10" [s $20^{\#} = 11.76 - 1.53 = 10.23\ \square''$
Use 2 - 10" [s $20^{\#}$ and $2\frac{1}{4}'' \times \frac{5}{16}''$ lacing bars.

Section U_{11} to U_{13}.

Maximum stresses compression – 114200# tension + 90200#
compression – 45100# tension + 45100#
compression – 159300# tension + 135300#

Required area = 159300 ÷ 13700 = 11.62 □"
Area 2 – 10" [s 20# = 11.76 □" Use
Use 2 1/4" x 5/16" lacing bars.

Section U_5 to U_8.

Maximum stress = + 96400
Required area = 96400 ÷ 16000 = 6.030 □"
Try 2 – 10" [s 15# Area = 8.92 – 1.53 = 7390 □" Use
Use 2 1/4" x 5/16" lacing bars.

Section U_5 to U_4.

Maximum stresses compression – 54300# tension + 61300#
compression – 27200# tension + 27200#
compression – 81500# tension + 88500#

$\frac{l}{r}$ = 58. P = 12000 81500 ÷ 12000 = required area = 6.80 □" Try 2 – 10" [s 15# Area = 8.92 Use
Use 2 1/4" x 5/16" lacing bars.

Section U_4 to U_1.

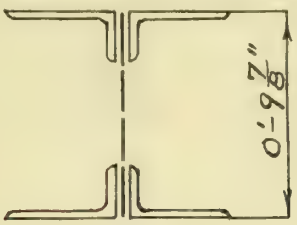
Maximum stresses compression – 28800# tension + 65000#
compression – 10800#
compression – 39600#

P = 10500 Required area = 39600 ÷ 10500 = 3.80 □"
65000 ÷ 16000 = 4.0 □"
2 – 10" [s 15# = 8.92 – 1.4 = 7.52 net area. Use
Use 2 1/4" x 3/16" lacing bars.

Design Of Web Members

The sections used in the design of web members will consist of either two (2) or four (4) angles laced.

Member U14 – L15



Maximum stress compression – 54700# tension + 48700#
compression – 24300# tension + 24300#
compression – 79000# tension + 73000#

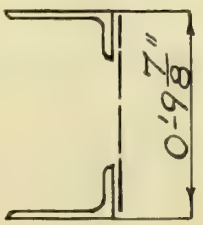
$l = 144"$ $r = 2$ $\frac{l}{r} = 72$ P 11000

Required area = 79000 ÷ 11000 = 7.20 □"

Area 4 – 4" X 3" X $\frac{5}{16}$" Ls = 8.36 □" Use

Use 2 $\frac{1}{4}$" X $\frac{5}{16}$" lacing bars.

It is customary in general practice to use $\frac{3}{4}$ of P for angles connected by one (1) leg. Use 12" X $\frac{5}{16}$" tie plates.



Member U14 – L14

Maximum stress compression – 29400#

$l = 60"$ $r = .89$ $\frac{l}{r} = 68$ $P = 11260 \times \frac{3}{4} = 8400$

Area = 3.5. Use 2 – 4" X 3" X $\frac{5}{16}$" Ls, 2 $\frac{1}{4}$" X $\frac{5}{16}$" lacing and $\frac{5}{16}$" X 12" tie plates.

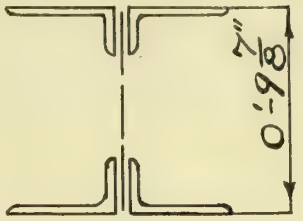
$U_{13}-L_{14}$ Maximum stress tension +45500#, $l = 144''$, $A = 2.84\ \square''$

$U_{13}-L_{13}$ " " compression -31900#, $l = 60''$

$U_{12}-L_{13}$ " " tension +32800#, $l = 144''$

$U_{12}-L_{12}$ " " compression -30000#, $l = 66''$

Use 2 - 4" x 3" x $\frac{5}{16}$" L_s, 12" x $\frac{5}{16}$" tie plates, 6" x $\frac{5}{16}$" stay plates about 3'-0" o.c. for members $U_{13}-L_{14}$, $U_{13}-L_{13}$, $U_{12}-L_{13}$, $U_{12}-L_{12}$.



$U_{11}-L_{12}$ Maximum stress compression -34000#. $l = 150''$

$r = 1.5$ $\frac{l}{r} = 100$ $P = 9000$ $A = 3.8$ Use 4 L_s 3" x 2½" x $\frac{5}{16}$"

Area = 6.48 $\square''$ Use lacing bars as above.

$U_{11}-L_{11}$ Maximum stress compression -33500# $l = 72''$

Use 4 L_s 3" x 2½" x $\frac{5}{16}$"

$U_{10}-L_{11}$ Maximum stress compression -22100# $l = 150''$

Use 4 L_s 3" x 2½" x $\frac{5}{16}$"

$U_{10}-L_{10}$ Maximum stress compression -33500# $l = 84''$

Use 4 L_s 3" x 2½" x $\frac{5}{16}$"

U_9-L_{10} Maximum stress tension +23400# $l = 156''$

$A = 1.4$ Use 2 L_s 4" x 3" x $\frac{5}{16}$"

U_9-L_9 Maximum stress compression -30900# $l = 106''$

Use 4 L_s 3" x 2½" x $\frac{5}{16}$"

U_8-L_9 Maximum stress compression -28900# $l = 168''$

Use 2 L_s 4" x 3" x $\frac{5}{16}$"

U_8-L_8 Maximum stress compression -27600# $l = 120''$

Use 4 L_s 3" x 2½" x $\frac{5}{16}$"

U_7-L_8 Maximum stress compression -14400# tension +30800#

compression -7200# tension +7200#

compression -21600# tension +38000#

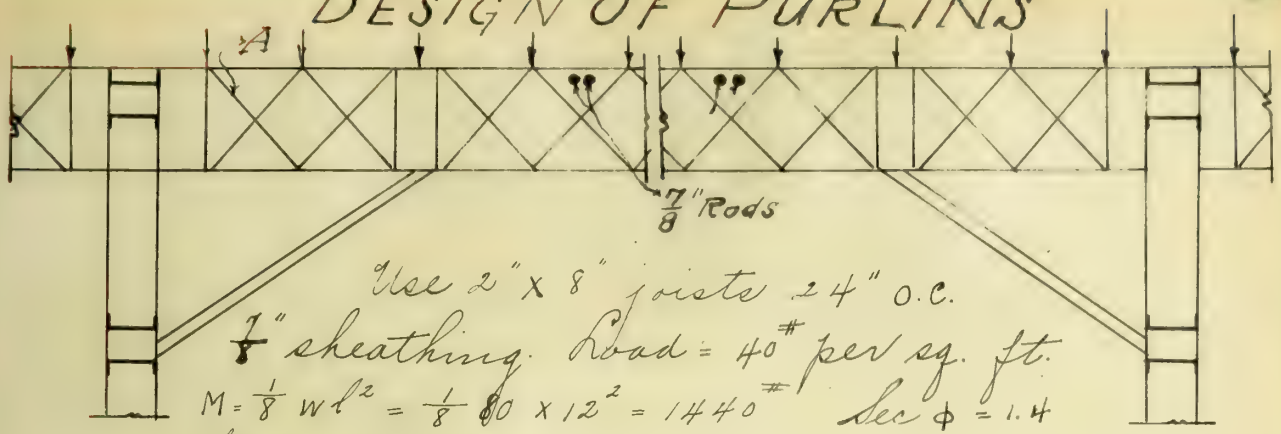
$l = 18.6$ $\frac{l}{r} = 120$ For compression 21600 ÷ 7600 =

Required area 2.8 $\square''$. For tension 38000 ÷ 16000 =

Required area 2.3 $\square''$. Use 4 L_s 3" x 2½" x $\frac{5}{16}$"

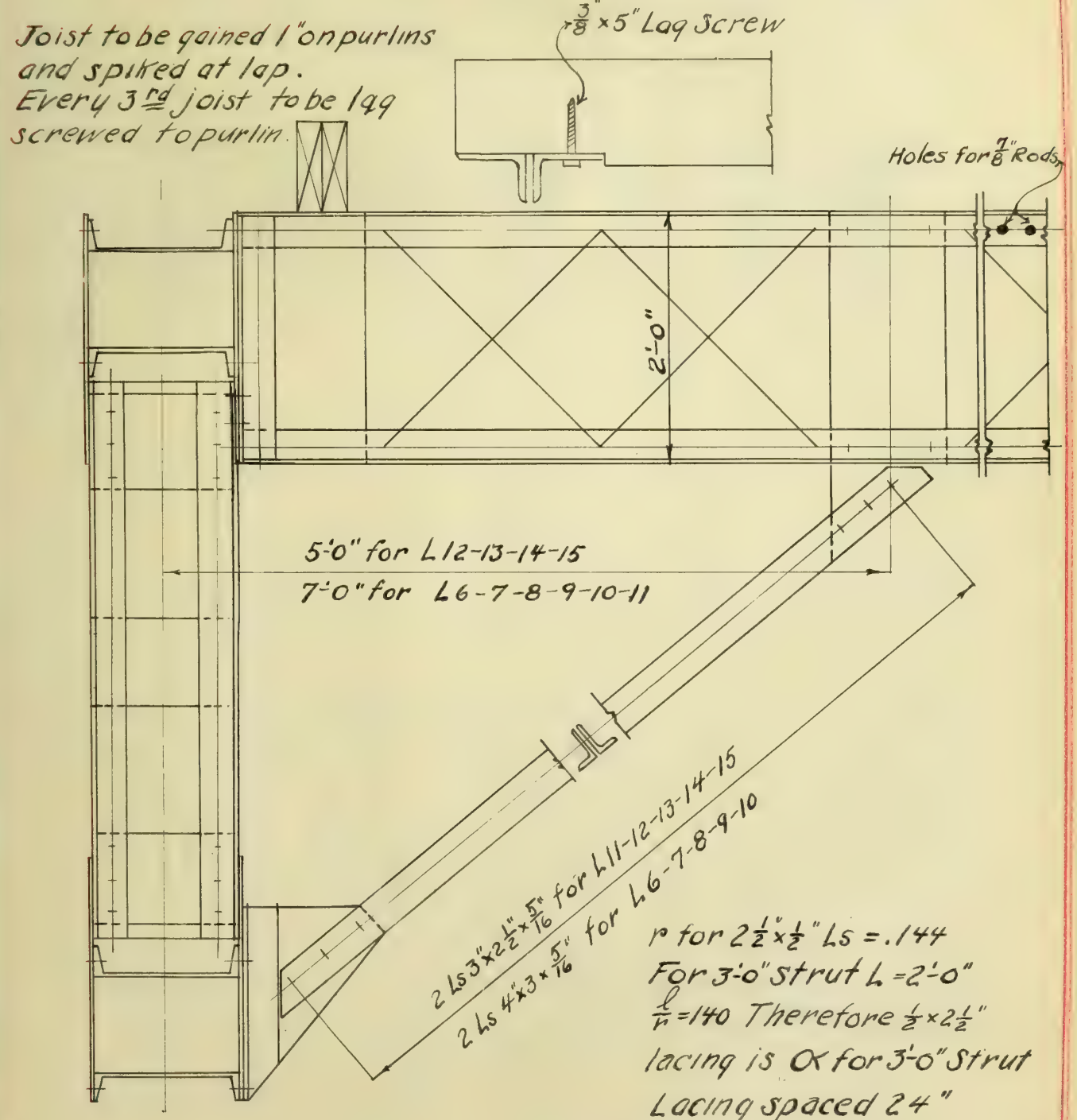
U_7-L_7 Maximum stress compression $-32500^{\#}$ $l = 156''$
$r = 2$, $\frac{l}{r} = 96$ Use 4 - 3" x 2½" x $\frac{5}{16}$" L_s. U_6 - L_7 Maximum stress compression $-31600^{\#}$ $l = 192''$ Use 4-4"x3"x$\frac{5}{16}$" L_s.
U_6-L_6 Maximum stress compression $-36400^{\#}$ $l = 180''$ Use 4 - 4" x 3" x $\frac{5}{16}$" L_s. U_5 - L_6 Maximum stress compression $-25600^{\#}$ tension $+31600^{\#}$
compression $-12800^{\#}$ tension $+12800^{\#}$
compression $-38400^{\#}$ tension $+44400^{\#}$
$l = 216''$ $r = 2$ $\frac{l}{r} = 108$ $P = 8400^{\#}$ per sq. in.
Required area $= 38400 \div 8400 = 4.57$ □"
Use 4 - 4" x 3" x $\frac{5}{16}$" L_s.
U_5 - L_5 Maximum stress compression $-27000^{\#}$ $l = 204''$
Use 4 - 4" x 3" x $\frac{5}{16}$" L_s.
U_4 - L_5 Maximum stress tension $+13400^{\#}$ $l =$
Use 4 - 3" x 2½" x $\frac{5}{16}$" L_s.
U_4 - L_4 Maximum stress compression $-12300^{\#}$ $l = 120''$
Use 4 - 3" x 2½" x $\frac{5}{16}$" L_s.
U_3 - L_4 Maximum stress compression $-13000^{\#}$ $l = 156''$
Use 4 - 3" x 2½" x $\frac{5}{16}$" L_s.
U_3 - L_3 Maximum stress compression $-7200^{\#}$ $l = 90''$
Use 4 - 3" x 2½" x $\frac{5}{16}$" L_s.
U_2 - L_3 Maximum stress compression $-47100^{\#}$ $l = 132''$
$r = 1.52''$ $\frac{l}{r} = 88$ $P = 9700$ $A = 4.8$ sq. in.
Use 4 - 3" x 2½" x $\frac{5}{16}$" L_s.
U_2 - L_2 Maximum stress compression $-32000^{\#}$ $l = 72''$
Use 4 - 3" x 2½" x $\frac{5}{16}$" L_s.
U_1 - L_2 Maximum stress compression $-76000^{\#}$ $l = 108''$
Use 4 - 4" x 3" x $\frac{5}{16}$" L_s.

DESIGN OF PURLINS



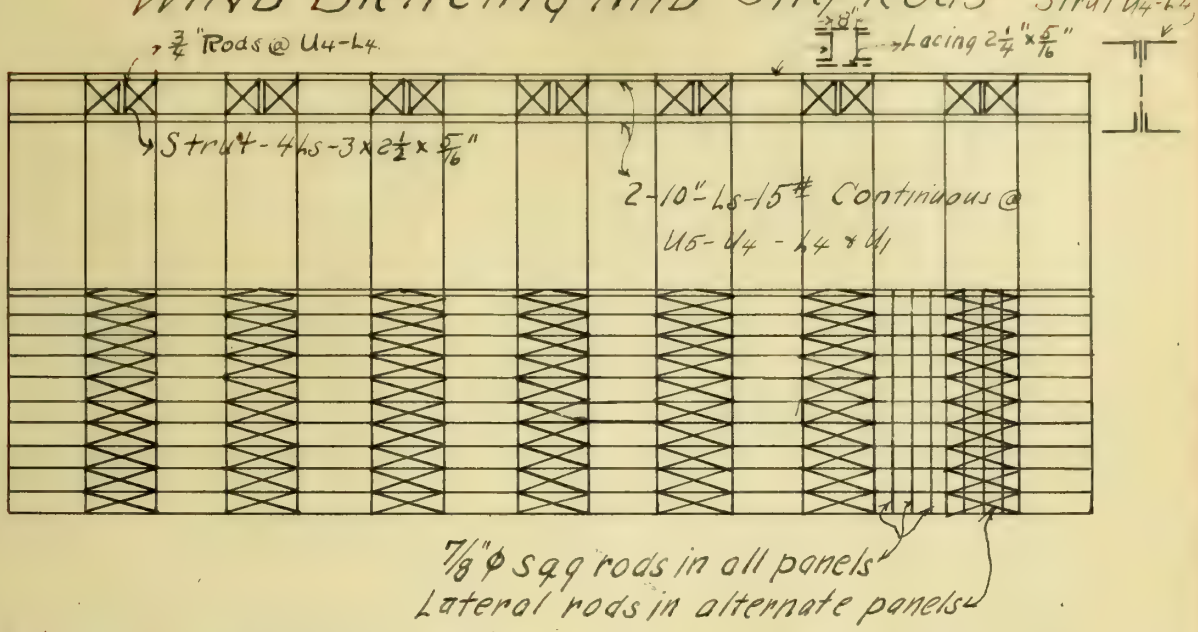
Use 2" x 8" joists 24" O.C.
1" sheathing. Load = 40# per sq. ft.
$M = \frac{1}{8} wl^2 = \frac{1}{8} 80 \times 12^2 = 1440^{\#}$ Sec $\phi = 1.4$
$\text{Stress} = \frac{1440 \times 12 \times 6}{2 \times 8 \times 8} = 810^{\#}$ per sq. in.
Concentrated load on one purlin = $40 \times 12 \times 2 = 960^{\#}$
Shear in bar A = $\frac{960 \times 5}{2} \times 1.4 = 3360^{\#} + 1344^{\#}$ (see below) $= 4700^{\#}$
Assume lacing $\frac{3}{8}" \times 2\frac{1}{2}"$: $r = .108$, $l = 16"$, $\frac{l}{r} = 150$, $P = 6000$.
Concentrated load = $960 \times 1.4 =$ stress $= 1344^{\#}$
Area one bar = .94□" Bar good for $.94 \times 6000 = 5640$
Lacing is therefore OK.
Concentrated loads are so close together we may figure the load uniform. $960 \div 2 = 480^{\#}$ per ft.
$M = \frac{1}{8} wl^2 = \frac{1}{8} \times 480 \times 27^2 = 43740$ ft. lbs.
The knee brace is neglected. As the snow will not stay on a steep roof, we will assume the worst thrust to be at 4 ⌀.
Sin ∠ 30° = .5 Load on side of strut = $480 \times .5 = 240$
$M = \frac{1}{10} wl^2 = \frac{1}{10} \times 240 \times 6.7^2 = 1070$ ft. lbs.
Assume top flange 2 - $3\frac{1}{2}" \times 3" \times \frac{5}{16}"$ L_s for 2 ft. struts ⊓⊓
Assume bottom flange 2 - $2\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{5}{16}"$ L_s for 2 ft struts ⊓⊓
Assume top flange 2 L_s $4" \times 3" \times \frac{5}{16}"$ for 3 ft. struts ⊓
Assume bottom flange 2 L_s $3" \times 2\frac{1}{2}" \times \frac{5}{16}"$ for 3 ft. struts ⊓
$\text{Stress} = \frac{1070 \times 12 \times 3.8}{12.5} = 3900^{\#}$
$43740 \div 1.9 = 23400 \div 3.86 = 6100^{\#}$
Maximum thrust on the building from wind pressure = $34000^{\#}$ This will effect both chords.
$34000 \div 6.8 = 5000^{\#}$ = wind thrust.
Roof thrust + $3900^{\#}$
Roof direct load + $6100^{\#}$
Wind thrust + $5000^{\#}$
Total stress top chord + $15000^{\#}$
Bottom chord $23400 \div 2.44 = 9600$
Section assumed is OK.

Detail Of Knee Brace
At
Panel Points


Joist to be gained 1" on purlins
and spiked at lap.
Every 3rd joist to be lag
screwed to purlin.
3/8" x 5" Lag Screw
Holes for 7/8" Rods
2'-0"
5'-0" for L 12-13-14-15
7'-0" for L 6-7-8-9-10-11
2 Ls 3" x 2 1/2" x 5/16" for L 11-12-13-14-15
2 Ls 4" x 3" x 5/16" for L 6-7-8-9-10
r for 2 1/2" x 1/2" Ls = .144
For 3'-0" strut L = 2'-0"
l/r = 140 Therefore 1/2 x 2 1/2"
lacing is Ok for 3'-0" Strut
Lacing spaced 24"
along strut.

DETAIL SHOWING

WIND BRACING AND SAG RODS



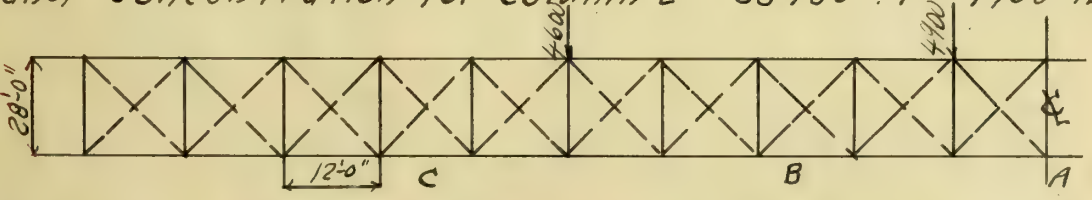
Diagonal rod bracing between U5 & U4 - U4 & U1 in some panels as roof bracing. Shear is same as from roof bracing.

DESIGN OF SWAY AND LATERAL RODS

Figure the purlins to take full Wind concentrated at Columns. Each set of sway rods will take 1/7 of Wind Load

Panel Concentration for Column 1 = 321720 ÷ 7 = 4600 lbs.

Panel Concentration for Column 2 = 33950 ÷ 7 = 4900 lbs.

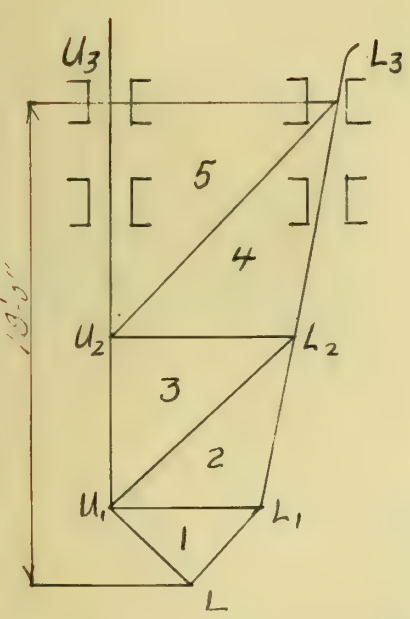


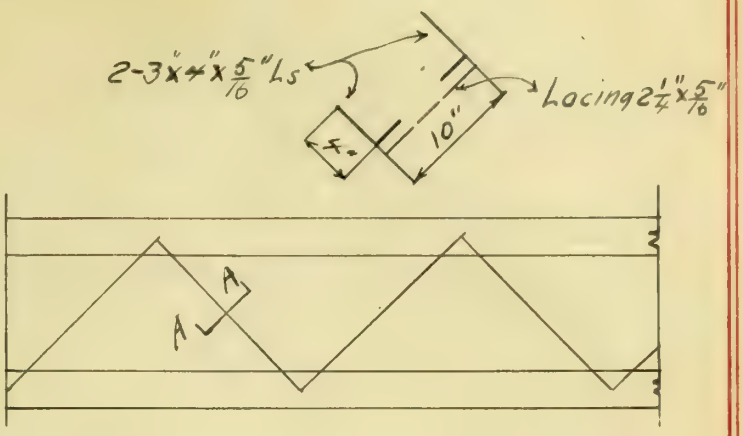
Sec ϕ = 1.09, 4900 × 1.09 = 5340#, 4600 × 1.09 5000#, Section A Shear = 0, Section B Shear = 4900#, Section B Stress in rod = 5340# Use ¾"ϕ Rods Section C Shear = 9500# Section C Stress in rod = 10340# Use 7/8"ϕ Rod.

Chord Stresses

R = 9500 Moment @ A = 9500 × 10 × 12 − 4900 × 12 − 4600 × 5 × 12 = 805200 ft lbs, 805200 ÷ 28' = 28700 ft lbs. As this is not 25% of average stress in chord - Will not increase chord section for wind.

DESIGN OF HORIZONTAL BRACING BETWEEN TRUSSES AT U3 & L3





CALCULATIONS

Wind Load at U_3 = 4750 lbs per truss

The horizontal bracing will transfer it equally to both Chords. The thrust at the pin equals 4750 lbs

Assume the arch fixed at the shoe.

Moment at U_3 h_3 = $4750 \times \frac{13}{2} = 30875$

Assume – 10"–35 # Ls

Stress = $\frac{30875 \times 12 \times 6}{4 \times 115.5} = 4000$ lbs

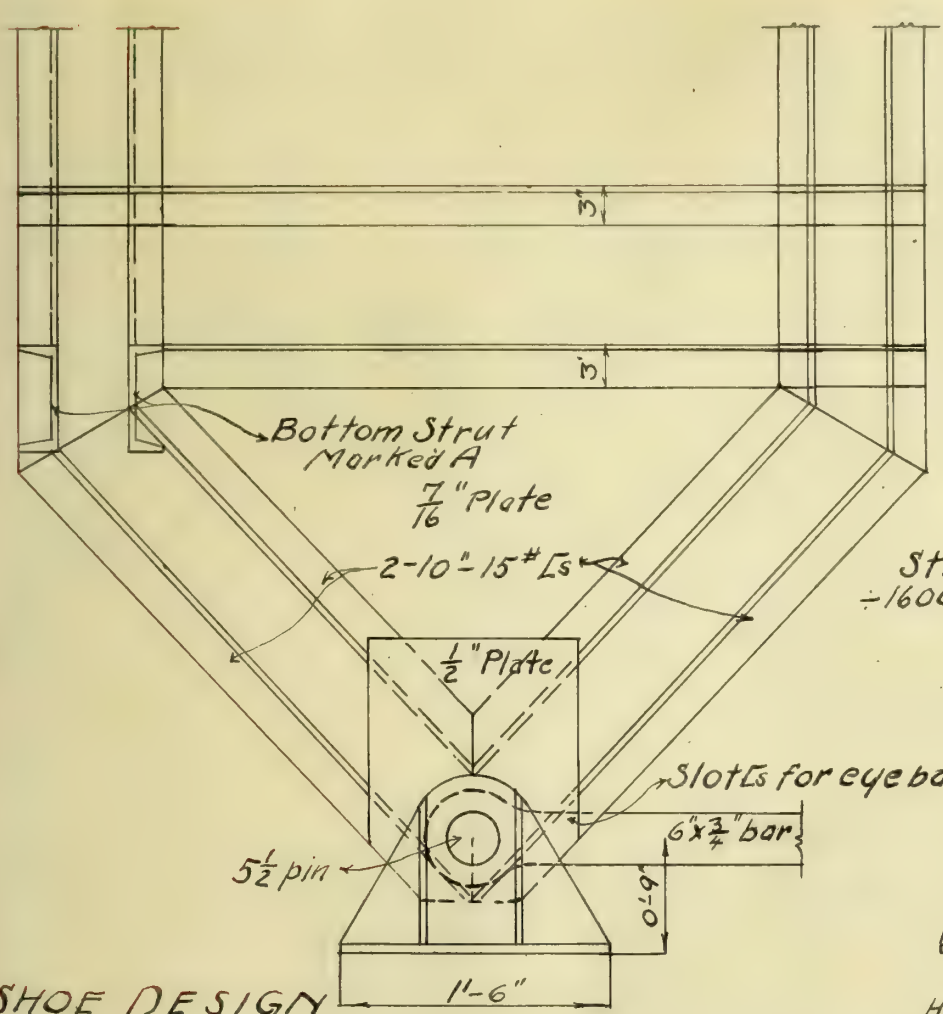
$h = .156$ $\frac{L}{r} = 42$ $P = 13000 - 4000 = 9000$

Stress h_2–h_3 $160400 \div 9000 = 17.8$ □" Area.

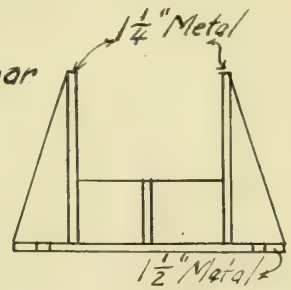
Use 2 Ls – 10"–35 # h_3 to h_1 Same for U_3 to U_2

No Strut below h_3

Detail Of Truss At Shoe Pin



Stress eye bar = 68400 ÷ 16000 = 4.3 □"



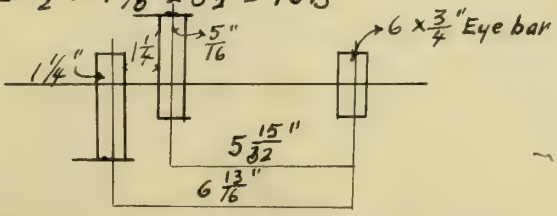
Holes for 1 1/4" Anchors

Shoe Design

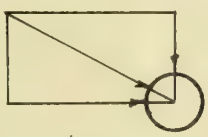
Vertical Load = 136800 lbs.
Horizontal Load = 68400 lbs
Shoe 136800 ÷ 400 = 341 □"
Make Shoe 24" long
Required width = 341 ÷ 24 = 14.2"
Make Shoe 18" Width
Make Shoe of 1 1/2" + 1 1/4" Metal

Pin Design

Allowable Unit Stress = 15000 #/□"
Req'd Area = 136400 ÷ 15000 = 9.1 □"
2" - 7/16" + 2" - 1/2 = 1 7/8" × 3 1/2" = 10.3



Allowable bending Stress = 20000 #/□"

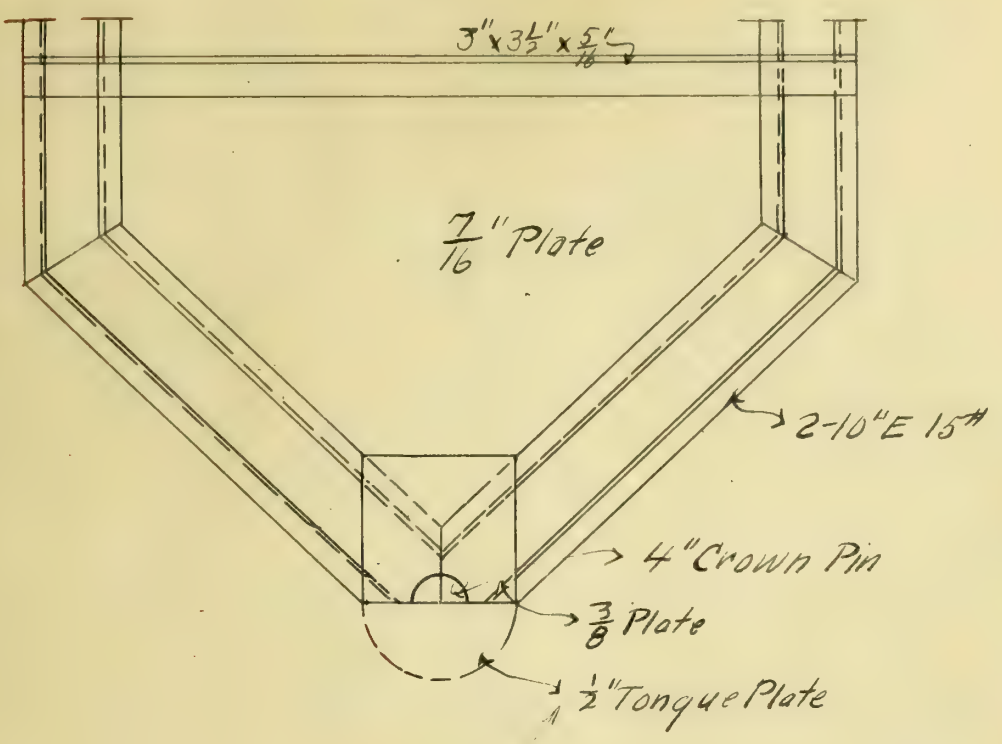


Horizontal Bending = 34200 × 5.47 = 187070
Vertical bending = 68400 × 1.34 = 91656
Maximum Moment = $\sqrt{187000^2 + 92000^2}$ = 20800 Use 5 1/2" Pin.

Bottom Strut Marked A

Shear in panel Above A = 9500. Load Coming to each arch leg = 4750
This is to be carried by bending to Shoe. Mom = 4750 × 4 1/2 = 20190 ft lbs
Stress = 20190 × 6/5 = 24230 . 7/16" plate is more than enough

DETAIL OF TRUSS AT CROWN PIN



Design Of Pin At Crown

Crown thrust = 68400 lb.

Assume Pin 4" Diameter. Allowable Unit Stress = 15000 #/□"

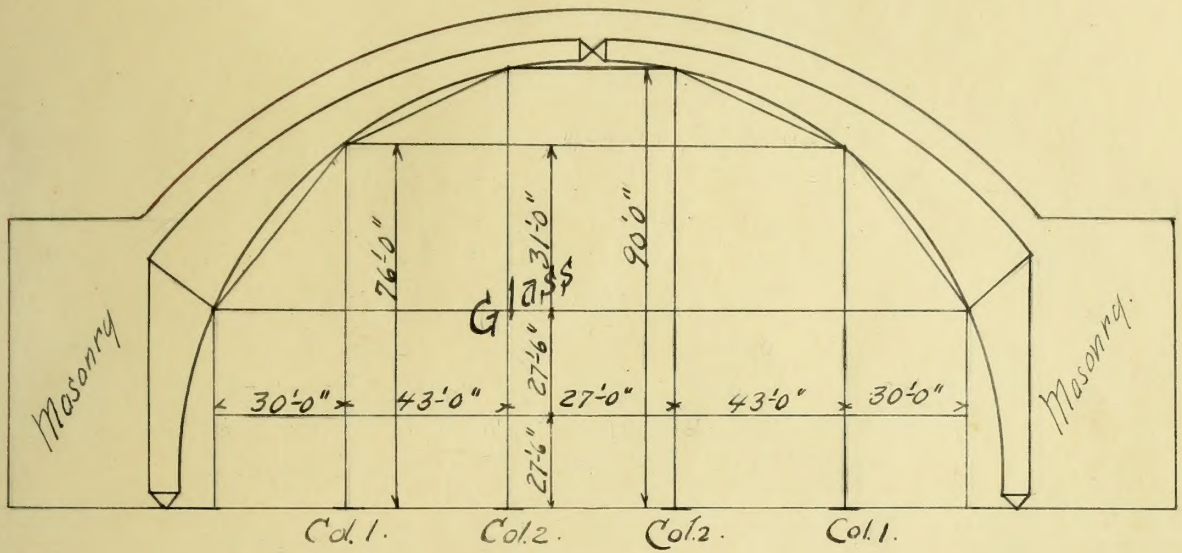
Bearing = 68400 ÷ 15000 = 4.56 □"

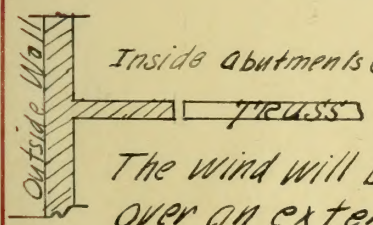
Area = 13/8 × 4 = 6.40 □" OK

Bending 34200 × 5/16 = 10700 " lbs

4 inch Pin is OK Use.

DESIGN OF STEEL COLUMNS AND FRAMING AT END OF BUILDING





Inside abutments @ ends of each truss.

END ELEVATION OF BLDG.

The wind will be assumed @ 20 #/□' As wind is not uniform over an extended surface We may take 20 #/□' as an average.

Load/□' of Col. 1 = 36.5 × 20 = 730 # Reaction = 730 × 44 = 32120

As wind stresses in sway bracing are small we will assume total end wind concentration on cols. Actually the wind on parapet would be taken at each panel point.

Make Cols 1 & 2 fixed top & bottom and inches back to back ls = 2'-6"

Load for Col 2 = 20 × 35 = 700 Reaction = 700 × 48½ = 33950

Make all columns same section as load are nearly the same.

Moment = $\frac{1}{8}$ × 700 × 31 × 12 = 260400 $S = \frac{MC^{(15)}}{I} = \frac{492000}{123.48} = 4000$

Wt. cu. ft. brick = 120 # Parapet Load @ Col 2 = 120 × 35 × 7 × 1 = 39400 #

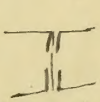
Parapet Load ÷ Area Col = $S' = \frac{39400}{13.72} = 2870$ #/□"

$S + S' = 4000 + 2870 = 6870$ #/□" = P

Try 4-6" × 3½" × ⅜" Ls 3'-6" apart laced.

$P = 16000 - 70\frac{\ell}{r} = 16000 - 70\frac{373}{3} = 7320$

Use 4-6" × 3½" × ⅜" Ls laced. for end Columns



For horizontal bracing use 4-2½ × 2 × $\frac{5}{16}$" Ls laced 3'-6" ~~c.c.~~ b to b.

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